ELKHEAD DAM/RESERVOIR
ENLARGEMENT HYDROLOGY REPORT

Prepared for

Colorado River Water Conservation District
201 Centennial
Glenwood Springs, Colorado 81602

1. INTRODUCTION

1.1 Background

Two recent reports, Yampa River basin study (Hydrosphere 1993) and a site-specific detailed feasibility study (Hydrosphere 1995) provide extensive documentation leading to a recommendation of enlarging the existing Elkhead Dam/Reservoir. The dam/reservoir is located on Elkhead Creek, a major tributary of the Yampa River. Liberal use has been made in this report of information provided in these two comprehensive documents by permission of the District and its consultants.

The existing dam/reservoir is located in Moffat and Routt Counties in northwest Colorado, as shown in **Figure 1.1**. The location of the dam is in the SW1/4 and SE1/4 of the SW1/4 of Section 16, Township 7 North, Range 89 West of the 6th Principal Meridian.

Station 0+00 on the original dam was off the east end of the dam (10,249.94 N and 49,502.64 E) approximately 2,000 feet east northeast of the southwest corner of Section 16; the new station 0+00 has not been established. This hydrology report is prepared in support of the design documents which will depict the raise of this dam approximately 43.5 feet to a height of 130 feet (elevation 6418.5) and to a storage of 44,900 acre-feet (at the service spillway, an elevation of 6406 feet).

1.2 Dam/Reservoir Classification

Since the dam will be raised to a height of 130 feet, the structure will be considered “large” by the Colorado SEO. The presence of downstream ranches and the town of Craig in the floodplain fringe means that loss of human life is possible in the event of a dam failure and that an SEO Class I hazard rating is applicable. In accordance with this criteria, the IDF or SEF will be the PMF. All three terms mean the same for the purposes of this report; the term PMF will be used for consistency. At this time, it is not planned to request a modification of this IDF requirement through performance of an incremental damages analysis.

Figure 1.1. Elkhead drainage area.

2. GENERAL DAM/SPILLWAY CONFIGURATION

2.1 Existing Project

The existing earthfill dam axis runs from southwest to northeast from the right abutment as shown in **Figure 2.1**. The dam has a concrete “duckbill” or “bathtub” service spillway with an ogee arch-shaped crest at elevation 6365 feet; the spillway is approximately 140 feet long at the left abutment interface. It has a capacity of approximately 16,000 cfs with the water level at the top of the dam, elevation 6375 feet. The service spillway has a 40-foot wide concrete chute which ends in a baffle block/riprap energy dissipation basin. A hydraulically-controlled 36-inch diameter primary outlet releases water from the reservoir at a location to the right of the spillway into the spillway chute. It has a capacity of approximately 180 cfs with the water level at the top of the dam, elevation 6375 feet.

The stage-capacity curve for the existing and proposed reservoir is shown in **Figure 2.2**.

2.2 Proposed Project

The existing dam will be raised 43.5 feet to a crest elevation of 6418.5 feet at the same location using downstream construction techniques. This requires construction of a new primary outlet, service spillway, and emergency spillway at the locations shown in Figure 2.1.

The concept for handling flood events at the enlarged Elkhead Dam/Reservoir consists of a primary outlet sized to handle emergency reservoir drawdown, deliver downstream flow demands, carry water to produce hydroelectric power (future facility), and pass frequent floods up to the mean annual peak flood without the use of a service spillway. It is not functionally satisfactory or economically feasible to provide a primary outlet which can handle more infrequently occurring flood flows. A service spillway and emergency spillway were therefore proposed to handle flows larger than the mean annual peak flood.

The intent of this conceptual approach of flow handling is to provide a primary outlet and service spillway which together control releases from the reservoir for all hydrologic events having a reasonable likelihood of occurrence during the 100-year design life of the dam/reservoir. In addition, in order to meeting State of Colorado and standard dam safety criteria, the dam must be able to withstand, without failure by overtopping, the statistically indeterminate PMF. An emergency spillway, located in an adjacent topographic saddle, was therefore proposed to assist in the conveyance of all flows exceeding the 100-year event. Since this emergency spillway directs rare flood flows to an adjacent drainageway which in turn will carry water back to Elkhead Creek 1 mile downstream, and since the drainageway must otherwise continue to function as it does now, the frequency and magnitude of water which it must handle under emergency conditions must be minimized. Therefore, a 3-foot surcharge pool (between the service spillway crest elevation of 6406 feet and the emergency spillway crest elevation of 6409 feet) has been provided within the reservoir to function together with the service spillway (neglecting the primary outlet) to handle most floods without using the emergency spillway. This combination of flow capacity and hydrograph attenuation reduces the frequency of use of the emergency spillway to approximately once in 500 years (based on rainfall flood frequency), but maintains a physical capability of handling extremely rare flows up to the PMF.

Figure 2.1. General plan of dam area.

Figure 2.2. Elkhead Reservoir area-capacity curve.

A 100-foot primary outlet tower, as shown in **Figure 2.3**, with a crest elevation of 6418.5 feet will be located in the reservoir approximately 260 feet north of the dam crest, as shown in Figure 2.1. The lowest elevation at which water will flow into the tower is 6333.5 feet. A 5-foot square sluice gate will exist at this elevation. Two additional sluice gates, each 3 feet square, will be located on different faces on the outside of the tower. The location of these gates will depend on the water temperature release requirements which remain to be determined. The gates have tentatively been evenly spaced between elevation 6333.5 and 6406.0 feet. All three gates will be guarded by trashracks.

A fourth 7.5-foot square gate (guard gate) will be located on the inside of the tower at the entrance to the penstock. The penstock will be a 5-foot diameter concrete-encased steel pipe and connected to the outlet tower at the elevation of the lowest inlet (6333.5 feet). The 760-foot long penstock will be located 200 feet west of the service spillway along the dam crest. Inside the tower, a 10-foot high bell-mouthed entrance will transition into the 5-foot diameter penstock. From the tower, the penstock will slope at 5.3 percent down toward the stilling basin located at the end of the spillway. The pipe will bifurcate before reaching the basin. The first branch will be a 15-inch diameter gated low-flow bypass. The second branch is to a butterfly valve followed by a Howell-Bunger™ valve. The purpose of the Howell-Bunger™ valve is to dissipate the water energy before it enters the stilling basin as well as to throttle the flow and bypass water from the hydroelectric facility. The 5-foot diameter penstock continues to the future turbine location. Until the hydroelectric facility is installed, an end cap will be used to keep flow from passing through this branch of the penstock.

The penstock was sized to meet the SEO requirement of having to lower the reservoir 5 feet in 5 days (considered the emergency reservoir drawdown), and to control water during the construction period. For the enlarged Elkhead Reservoir, 5,500 acre-feet of water needs to be released during the 5-day period. To meet this requirement, a 5-foot diameter penstock and 48-inch diameter Howell-Bunger™ valve, which controls the maximum rate of flow (675 cfs), is proposed.

The purpose of the service spillway is to release flows which exceed the maximum operating capacity of the primary outlet structure and to route floods to the downstream natural channel. As shown in **Figures 2.4 and 2.5**, flow enters the spillway across an apron, then discharges over a 100-foot wide rectangular ogee crest, down the rectangular-shaped concrete spillway and into the stilling basin before being delivered to the natural channel of Elkhead Creek. The spillway has been configured to handle approximately the unattenuated 100-year snowmelt flood with the reservoir water level at the top of the 3-foot surcharge pool. It will also assist in passing the PMF event in combination with the emergency spillway.

A smooth transition will be provided for water to flow into the spillway. Quarter-circle shaped vertical concrete walls were configured at the entrance with the top of the walls tapering down from an elevation of 6418.5 feet at the dam crest to approximately 6403.3 feet at the toe of the apron.

The floor of the entrance out to the radius of these walls will be constructed of concrete. Riprap will be provided at the entrance toe on the banks surrounding the entrance up to the dam crest to protect this area from potential scour. No riprap will be needed on the apron that is excavated into bedrock or where the water velocities do not exceed 5 feet per second (fps).

Figure 2.3. Primary outlet plan and profile.

Figure 2.4. Service spillway plan.

Figure 2.5. Service spillway profile.

The crest of the service spillway will be at an elevation of 6406 feet. The shape of the crest will be of the ogee type. As shown in Figure 2.5, the upstream face of the crest was configured with a 6-foot vertical face extending up from the apron floor. The ogee-shaped crest will be constructed down to elevation 6398 feet to ensure that the crest will function properly without being affected by flow in the spillway chute.

An important criterion for the service spillway and apron was that they be located on the bedrock in the left abutment. This provides stability for the spillway as well as erosion protection at the base of the spillway. Along with this criterion, it was important to minimize the spillway construction cost by minimizing the amount of excavation and keeping the length of the spillway as short as possible. This was accomplished by configuring the spillway to conform with the topography while remaining founded in the left abutment as shown in Figures 2.1 and 2.5. A spillway chute with an initial slope of 7:1 gradually changing to 3:1 was found to best meet these requirements. With this configuration, it was determined that the length of the spillway would be approximately 560 feet.

The relatively steep slopes of the service spillway chute will cause the flow in the spillway to be supercritical. Freeboard will be needed to provide for wave action, flow, bulking, splash, and spray. The required freeboard was calculated by using an empirical equation which gives a reasonable indication of the desirable freeboard. It was concluded that the wall height be 7 feet along the entire length of the spillway to meet the freeboard requirement.

A stilling basin will be provided to dissipate the energy of flowing water carried by the service spillway. The purpose of the stilling basin is to dissipate the kinetic energy of the falling water to a level equal to the natural energy gradient of Elkhead Creek. The energy of flow from the primary outlet, which is discharged via the hydroelectric facility and/or Howell-Bunger™ valve to the stilling basin, is dissipated separately as part of the power production or the valve characteristics.

Hydraulic computations from service spillway analysis determined the size and type of the stilling basin. The type of stilling basin chosen, based on the shallow depth, high velocities, and high Froude Number of the design flow, was USBR Type II. In a Type II design, there are chute blocks at the entrance of the basin and a dentated sill at the downstream end. The configuration of the basin was determined using the USBR's Monograph 25 (USBR 1978). The stilling basin size will be 100 feet wide (the same as the service spillway) and 45 feet long. A diagram of the basin is presented in **Figure 2.6**.

In order to take maximum advantage of the site topography, the emergency spillway is proposed to be located in an adjacent topographic saddle near the dam embankment on the west end of the reservoir as shown in Figure 2.1. The floor of the spillway is set at elevation 6409 feet in order to be founded into the underlying sandstone bedrock as described in the preliminary geotechnical investigation (Woodward-Clyde 1994). Discharges from the emergency spillway will be directed away from the dam embankment towards a drainage swale along Moffat County Road 29. From there, the water will be carried in the drainage swale for approximately 1 mile before discharging back into Elkhead Creek approximately 1.5 stream-miles below the dam.

Figure 2.6. Stilling basin.

A 3-foot flood surcharge pool is proposed within the reservoir to enable the primary outlet and service spillway to function during normal reservoir operation and to control releases from the reservoir for most floods without using the emergency spillway. The crest of the service spillway is therefore proposed to be set at elevation 6406 feet to provide approximately 3,300 acre-feet of surcharge storage in the reservoir. A bottom width of 350 feet is recommended for the emergency spillway. A cross section of the emergency spillway is shown in **Figure 2.7**.

Figure 2.7. Emergency spillway section.

3. DRAINAGE BASIN CHARACTERISTICS

3.1 Subbasin Breakdown

The 205-square mile drainage basin was further divided into subbasins for the purpose of hydrologic modeling. This is to enable more site-specific characterization of basin parameters and the computation of a natural composite basin hydrograph responsive to those characteristics versus a standardized unit graph for the entire basin. Five subbasins which vary in size from 21 to 68 square miles, measured from best available mapping (Figure 1.1) were configured to closely address as many of the following criteria as possible:

1. Areas monitored by streamflow gages
2. Major tributaries
3. Variance in aspect
4. Consistent drainage area size of approximately 50 square miles to facilitate meteorologic storm positioning
5. Land use and ground cover consistency
6. Soils consistency
7. Ability to combine effectively into a modeling network
8. Topography consistency

As a result, the North Fork and California Park subbasins were delineated representing the undeveloped high elevation lands in public control. The Calf Creek subbasin is a transition to the Dry Fork and Long Gulch subbasins, which represent the lower basin lands in private agricultural use.

3.2 Geology/Soils

A significant amount of information exists on the various soil types and geology of the drainage basin. The U.S. Forest Service (USFS) has mapped the soil types in the National Forest under their control and the Soil Conservation Service (SCS) has mapped the remaining soil types in both counties. Private mineral resource investigations have also been conducted, but these are not generally available. In addition, a detailed, composite geotechnical evaluation has been conducted of the dam raise site and nearby construction material sources (Woodward-Clyde 1994). Unfortunately, little of this information is published or compiled in a conveniently useable form for hydrologic modeling purposes. A summary of relevant information was gleaned from these published and unpublished sources and is described by subbasin in the following sections.

3.2.1 North Fork Subbasin

The North Fork subbasin is mapped by the USFS and SCS. The soils are generally deep, well-drained stony loams with some clay as characterized by Passar-Cochetopa-Doughspon and Leaps-Rhone complex, which make up hills, mountainside and plateau landforms. The hydrologic soil group is C (refer to Section 4.6 for discussion of different soil groups).

3.2.2 California Park Subbasin

The California Park subbasin is mapped by the USFS and SCS. The soils are generally deep, well-drained and coarse as characterized by Northwater-Skylick loams and Foidel-Clayburn-Dranyon which make up hills, mountains, and plateau landforms. The hydrologic soil group is B.

3.2.3 Calf Creek Subbasin

The Calf Creek subbasin is mapped by the SCS. The soils are generally deep, well-drained and fine as characterized by Binco-Gourley and Cochetopa-Routt-Gothic which make up hills, side slopes, and mountain landforms. The hydrologic soil group is C.

3.2.4 Dry Fork Subbasin

The Dry Fork subbasin is mapped by the SCS. The soils are generally deep, well-drained and fine as characterized by Passar-Cochetopa-Doughspon and Binco-Gourley which make up hills, side slopes, and mountain-side landforms. The hydrologic soil group is C.

3.2.5 Long Gulch Subbasin

The Long Gulch subbasin is mapped by the SCS. The soils are generally deep, well-drained and fine as characterized by Hesperus-Evanot-Emlin which make up side slopes, benches, and terrace landforms. The hydrologic soil group is B/C.

3.3 Vegetation

Vegetation varies significantly across the drainage basin. The variance is in response to elevation and the attendant corresponding changes in soil, moisture, and temperature. The elevation ranges up to 7500 feet and is dominated by dry land wheat agriculture, unirrigated pasture/rangeland, and sagebrush shrublands in upland areas with narrow riparian cottonwood woodlands in bottomland areas. Mixed mountain shrubs transition to aspen woodland and brush/grass meadows from 7500 to 9000 feet in the National Forest. Ponderosa pines become more common around 9000 feet continuing up to sparse, mixed subalpine vegetation of the mountain peaks.

Ground cover is generally good except in unplanted or alternate fallow wheat strips, and small areas or rock slopes, roads, and human activity. Agriculture and ranching activity has significantly modified the native ground cover, and in some areas, this contributes substantially to sediment loads in Elkhead Creek.

3.4 Topography

The existing dam and reservoir are located on Elkhead Creek approximately 10 miles northeast of the town of Craig and 3 miles upstream of the confluence with the Yampa River in the northwest corner of Colorado. The Elkhead Creek headwaters are along the Wyoming border in the Elkhead mountains which contains Bear Ears Peak as the distinguishing land feature. Elkhead Creek flows from a subalpine environment of moderate density pine forest at an elevation exceeding 10500 feet to a sagebrush/grass-dominated environment of rolling hills at elevation 6300 feet at the dam site. The drainage basin area above the dam is 205 square miles with water collected by the major tributaries of Dry Fork Elkhead and North Fork Elkhead Creeks. High altitude areas within the basin accumulate a significant snowpack which runs off in the spring.

Steep, 10 to 40 percent slopes, exist in the high altitude areas above 9000 feet. These moderate to 1 to 3 percent at the dam site, with locally steeper areas. The topography ranges from exposed, steep bedrock to a moderately sloped, lush mountain park known as California Park to soil-covered rolling hills with gently sloping small mesas at the dam site.

3.5 Land Use

The historic and existing land use in the project area has been largely dictated by climate and natural resources. Archaeological evidence indicates only sparse occupation of the area by native people prior to European settlement, following initial Spanish exploration of the area in 1776. European activity through the 1870s was limited to modest trapping, exploration, and livestock-raising activity. Communities such as Craig (established in 1889) developed around cattle, sheep, and speculative homesteading industries, but the area remained sparsely populated due largely to the lack of adequate transportation facilities. The railroad in the 1890s and U.S. Highway 40 in 1920 opened up the area, but it continued to be characterized by a relatively slow population growth.

Routt National Forest was established in 1905. A fledgling recreation and ski industry began in the late 1920s, but did not fully develop until the early 1960s and never really had a significant impact in the Craig area. Uranium, oil, and coal all saw periods of exploration and development through World War II. Coal, in particular, received significant attention later during the mid-1970's energy crisis when the Craig and Hayden coal-fired generating stations were built. The local coal resources and generally remote location stimulated this development. The area remains today a reflection of this development history with dryland farming, livestock ranching, coal mining/power generation, and recreation dominating the existing land use. Existing land use in the Elkhead Creek basin is rural consisting of approximately 50 percent low density, private farming and ranching activity, and 50 percent undeveloped National Forest public land.

The existing Elkhead Dam/Reservoir have done little to change general land use in the vicinity since it was constructed in 1974. Beside the obvious replacement of native riparian and upland vegetation with a reservoir, the adjacent land use remains largely the same as it was 20 years ago. Public recreational activity in the area is best characterized as light day use.

Future land use in the Craig area will be largely controlled by the level of mining and energy-based activity and secondary effects. Urbanization, population growth, modifications to transportation systems, increases in outdoor recreation, and related land-use changes will occur, but will probably be minor. Within the Elkhead Creek catchment itself, future land use is not expected to change significantly from that of current conditions.

A larger reservoir will encourage more general recreational use by area residents, but is unlikely to change from day-use to a "destination location." If private ownership of reservoir perimeter lands was to be allowed, some recreation home development would occur, but even this impact is unlikely to extend significantly landward from the reservoir edge due to the generally semiarid and treeless nature of the adjacent lands. Existing and future land use of the catchment is expected to be effectively the same.

3.6 Meteorology

A search for officially gathered and private meteorologic information within the Elkhead drainage basin determined that information does not exist. That is, there is no basin-specific information on precipitation, temperatures, wind speed, solar radiation, areal snow cover, snowpack water equivalent, and others. Observations indicate that a significant snowpack accumulates annually in high altitude areas of the basin. Moderate intensity/moderate duration rain in combination with snowmelt produces the annual peak streamflows which occur in late spring. Summer thunderstorms are very common, but they seldom produce significant rain; when rain occurs, it is intense and short in duration. Refer to the streamflow portion of this document for a further, indirect indicator of precipitation characteristics.

Some meteorologic information exists outside the drainage basin, but it was not used directly. For instance, it was used indirectly as a part of published regional information which exists in the NOAA Atlas for Colorado (NOAA 1973). A more complete description of site meteorology is made in the "Site-Specific Probable Maximum Precipitation Study of the Elkhead Drainage Basin" report (NAWC 1996), which is included in **Appendix A**.

3.7 Streamflow

Streamflow within the drainage basin is well documented by four USGS stream gages, one of which remains in operation. Three of these gages recorded flows upstream of the existing reservoir:

1. gage 9245000: Elkhead Creek near Elkhead
2. gage 9244500: Elkhead Creek near Clark (the only currently active gage)
3. gage 9245500: North Fork Elkhead Creek near Elkhead

A fourth gage was located immediately upstream of the confluence with the Yampa River (gage 9246500). This gage monitored flows from 1910 though 1918. Its record is short and contains suspect daily flow information and, as a result, was not of particular value. These gages indicate that at the dam, the annual peak flow is approximately 600 cfs, occurring around May 20 as a result of snowmelt and spring rains. Flow decreases to less than 10 cfs by late in the year. Occasional summer thunderstorms produce some short periods of increased streamflow.   In the absence of any meaningful meteorologic information and little other quality physical data, the streamflow information was the basis for a significant part of this hydrologic study.

Log Pearson Type III statistical streamflow analyses were conducted for the three upstream gages to form the basis for flood peak projections. One particularly notable observation made from the streamflow records was that no annual peak flows occurred outside of the mixed event, snowmelt-dominated period of April through June. In fact, the highest flows for any rainfall event outside of that period, including the rainfall-dominated (thunderstorm) months of July through September, were a full order of magnitude less than the mean annual flood.

Therefore, the statistics used for the annual flood frequency analysis of all events is the same as events limited to the snowmelt-dominated months of April through June. This information indicates a clear domination of snowmelt (and mixed event) floods versus rainfall-only floods. A more precise separation of rainfall events from snowmelt events was not possible due to the aforementioned lack of meteorological information. This inability to separate rainfall events from combined events and knowing that no rainfall-only events are represented in the annual flood peaks resulted in complete reliance on the NOAA Atlas for Colorado (NOAA 1973) for rainfall information.

4. HYDROLOGIC ANALYSIS

4.1 General Approach

The rural nature of the watershed, lack of meteorological information, and availability of streamflow information were the controlling characteristics used to establish the basic hydrologic methodologies.

It was decided to use regional rainfall information in combination with basic physical characteristics to compute floods from rainfall (for primarily rainfall events) in a “design storm” analysis.

Adequate information exists to estimate primarily snowmelt flood frequency from the streamflow records previously described. This information is then included as baseflow, in accordance with the criteria of the SEO and standard practices of the USBR, as applicable in the modeling of PMF events, and as a distinct flood-producing phenomenon for frequency-based flood events. For PMF events, the 100-year snowmelt flood peak was added as baseflow to rainfall events occurring during the period of May 1 through June 15, and the mean annual snowmelt flood was added during April and June 16 through 30. Computer model HEC-1 (USACOE 1990) was used to model basin hydrology as it allows snowmelt to be considered as baseflows (when applicable), has several unit graph, channel pool, and level pool options, and other modeling flexibility.

4.2 Unit Hydrographs Sensitivity Analysis

Insufficient site data exist for Elkhead Creek to construct a family of basin-specific unit hydrographs. Fortunately, several standardized regional techniques are available which can be applied to the Elkhead drainage basin. Three synthetic unit graph procedures are in common use on nearby similar projects in Colorado. A sensitivity analysis of these methods was conducted to determine the unit hydrograph procedure to be used for the hydrologic computations in the Elkhead basin. The three unit graph methods are the SCS, USBR, and Snyder methods. The sensitivity analysis was conducted in order to identify which method provides the most stressing runoff hydrograph in keeping with the need to maximize flood runoff in computation of the PMF. Each of the three methods requires different characteristic parameters. A description follows of these characteristics and how they are used as model input parameters for each method.

4.2.1 SCS Method

The SCS method (SCS 1984) uses only the basin lag time in order to compute a basin unit graph. The SCS unit hydrograph is computed using dimensionless unit hydrographs that are based on extensive analysis of measured data. It is based on the evaluation of unit hydrographs for a large number of actual watersheds which were then made dimensionless. An average of these dimensionless unit hydrographs was computed. The time base of the dimensionless unit graph is approximately 5 times the time to peak and approximately 3/8 of the total volume occurred before the time to peak.

The lag time (L) is computed as 0.6 times the time of concentration (T_c) and the time of concentration for the purposes of the sensitivity analysis, is defined as:

$$T_c = \frac{D^{1.15}}{7700H^{0.38}} \qquad (4.1)$$

where D = length measured along the watercourse from the location of interest to the most distant point in the watershed, feet

H = elevation difference along flow path, feet

The model then uses the entered lag time to compute the time to peak (T_p):

$$T_p = 0.5\ Dt + L \tag{4.2}$$

where Dt = duration of excess, hours, and ≤ 0.29*L

The peak flow (q_p)of the unit hydrograph (cfs/in) is defined as:

$$q_p = \frac{484\ A}{T_p} \tag{4.3}$$

where A = basin area, square miles

4.2.2 USBR Method

The USBR method uses the USBR lag time and dimensionless discharge, which is a function of the lag time and unit runoff in cubic feet per second (ft^3/s) days. The dimensionless discharge was determined by using Table 4.9 of the “Flood Hydrograph Manual” (USBR 1984), and the computation procedure in “Design of Small Dams” (USBR 1987).

The USBR method uses the dimensionless unit hydrograph technique which requires determination of the unit duration of the synthetic unit hydrograph. The unit duration approximates the lag time divided by 5.5. The dimensionless unit hydrograph is expressed in terms of time in percent of lag time plus the semiduration of unit rainfall. The USBR has developed general and thunderstorm dimensionless unit hydrograph data for many regions. The relationship most appropriate for the Elkhead basin is the one for the Rocky Mountains.

4.2.3 Snyder Method

The Snyder method uses the Snyder standard lag and peaking coefficient. The formation of the unit hydrograph for the Snyder method depends on the time to peak, peak discharge, time-base duration of the rainfall excess, and width of the unit hydrograph at both 50 and 75 percent of the peak discharge. The time of the peak depends on the duration of the rainfall excess and lag time. Snyder relates the lag time to the length of the main channel, a watershed-shaped parameter (centroid of the basin), and watershed storage coefficient.

4.2.4 Selected Unit Hydrograph Procedure

The SCS and Snyder methods are more nationwide methods; the USBR method is considered more regional. In other related studies, the USBR method was used by Harza (Harza 1991) and the Division of Wildlife (DOW 1984) for the Elkhead basin, and the Snyder method was used by Boyle Engineering Corporation on Wolford Mountain (Boyle 1991, 1995). The SEO suggests the use of the USBR’s “Design of Small Dams” (USBR 1987) hydrograph procedure. It is interesting to note that Harza’s report (Harza 1991) used the USBR unit hydrograph procedure, but used the SCS runoff coefficients, and that the USBR lag time computation is a version of the Snyder method computation.

Three models were configured for the 100-year hydrology, differing only in the unit graph procedure. It was discovered that the SCS method produced the highest peak rate of flow and the Snyder method the lowest, but all produced relatively close results. The SCS unit graph procedure was therefore adopted for use on all the models of this study; both PMF and frequency-based floods.

4.3 Precipitation

Due to the lack of Elkhead basin precipitation information, published regional information (NOAA 1973) was used for the rainfall amounts for the defined frequency-based floods up to the 100-year event. The SCS standard duration of 24 hours was used for all events, together with a Type II storm temporal distribution and uniform areal rainfall. The 500-year rainfall amount was estimated by a straight line extension of the known rainfall amounts on log-probability paper. The rainfall amounts shown in **Table 4.1** are the basin-averaged base rainfall amounts with drainage area adjustment applied as described in the Atlas (NOAA 1977).

Table 4.1. Elkhead Basin Precipitation.

Frequency (-year)	Gross Rainfall (inches)	Adjusted Rainfall
2	1.36	1.26
10	1.94	1.80
100	2.86	2.66
500	3.48	3.24

4.4 Lag

Lag time is a hydrologic characteristic which is a measure of the time from the center of mass of excess rainfall to the peak rate of runoff. The lag time is a watershed parameter that is often related to the time of concentration; the travel time of water from the hydraulically most distant point to the point of interest. The time of concentration can be estimated from watershed characteristics such as watershed length, slope, and flow retardance. Based on many storm events, the SCS has established a relationship between lag and time of concentration as $L = 0.6*T_c$ for watersheds with average natural conditions and uniform distribution of runoff.

Time of concentration was computed for subbasins individually using velocities of overland and channelized flow components. The corresponding composite time of concentration was also computed using several generalized algorithms adjusted for the basin land use. The lag was computed directly using several generalized algorithms as well. The resulting values compared very closely. As a final check, these computed values were compared against lags for other similar areas, including for the nearby Muddy Creek basin (Boyle 1991, 1995), and found to be very close.

In this analysis, the length and slope were determined by using USGS 7.5-minute quadrangle maps of the area and retardance was estimated based on standard relationships.

Table 4.2 shows the adopted lag and drainage for the entire basin and each subbasin.

Table 4.2. Basin and Subbasin Lag Times.

Basin	Basin Area (mi^2)	Lag Times (hrs)
North Fork	21.5	1.24
California Peak	67.7	2.76
Calf Creek	21.3	1.91
Dry Fork	41.6	2.47
Long Gulch	52.9	2.19

4.5 Antecedent Moisture

The initial soil-moisture condition, or antecedent moisture condition, is a parameter usually expressed as a composite index which can be approximately representative of the initial soil-moisture condition and can be easily measured. Some of the characteristics represented by the index are groundwater flow at the beginning of the storm, basin evaporation, and antecedent precipitation.

Antecedent soil moisture is known to have a significant effect on both the volume and rate of runoff and because of this, SCS developed antecedent soil moisture conditions I, II, and III. Soil condition I pertains to those soils that are dry, but not to wetting point. Soil condition II is used for average conditions, and soil condition III pertains to saturated soils (i.e., those subject to heavy rainfall or light rainfall and low temperatures having occurred within the last 5 days). Most curve number (CN) values (refer to following section description) are for the average antecedent moisture condition (AMC II). When the conditions are below average (AMC I) or above average (AMC III), the CN values must be adjusted. The AMC II for November through March is reflected by 0.8 inch of precipitation and that for April through October is 1.75 inches.

In the Elkhead basin hydrologic analysis, two conditions were possible in the analysis of the PMF event. It was determined that for “design storm” rainfall events occurring during April, May, or June, AMC III would be used in the analysis. The reason for this was that during this time of the year, it is possible that snow will still be on the ground, the ground frozen, or the soil saturated from snowmelt runoff. This high antecedent moisture condition increases the amount of rainfall runoff. During all other months, AMC II was used.

4.6 Runoff

Runoff is produced from rainfall through the use of a loss rate function, which for the SCS unit graph technique, is described as a CN function. The CN is an index that represents the combination of a hydrologic soil group, determined from soil classification, and land use and treatment classification. The CN shows the relative value of the hydrologic soil-cover complexes in producing direct runoff. Soils can be classified according to their hydrologic properties. There are four major soil groups used for classification of soils:

Group A = Soils with low runoff potential/high rate of water transmission

Group B = Soils having moderate rate of water transmission

Group C = Soils having a slow rate of water transmission

Group D = Soils with high runoff potential/very slow rate of water transmission

Cover is any material (usually vegetation) covering soil and providing protection from the impact of rainfall. Often, land use as an index of the cover conditions is used in hydrologic analysis rather than detailed information about the cover. As previously mentioned, the Elkhead basin was subdivided into five subbasins. For each of these basins, a soil-cover complex was determined.

North Fork. Soil was determined as being type C and the cover 35 percent sagebrush/grass (100 percent good condition), pine (60 percent), aspen forest (75 percent good condition), and dryland agriculture and farms (5 percent fair condition). CN is 63.

California Park. Soil was classified as being type B and the cover 50 percent sagebrush/grass (100 percent good condition), pine (45 percent), aspen forest (75 percent good condition), and dryland agriculture and farms (5 percent fair condition). CN is 51.

Calf Creek. Soil group was classified as being type C and the cover 65 percent sagebrush/grass (100 percent good condition), pine (15 percent), aspen forest (75 percent good condition), dryland agriculture (15 percent fair condition), and farms (5 percent fair condition). CN is 66.

Dry Fork. Soil group was classified as type C and the cover 75 percent sagebrush/grass (100 percent good condition), pine (20 percent), aspen forest (75 percent good condition), and dryland agriculture and farms (5 percent fair condition). CN is 63.

Long Gulch. Soil group was classified as between types B and C and the cover 75 percent sagebrush/grass (100 percent good condition), dryland agriculture (20 percent fair condition), and farms (5 percent fair condition). CN is 61.

4.7 Model Configuration

Based on the description of hydrologic parameters described in the preceding sections, a HEC-1 (USACOE 1990) hydrologic model was configured. The basic characteristics of that model remain unchanged for analysis of all frequency events except for baseflow and antecedent moisture associated with PMF events, as previously described.

Subbasin hydrology was computed individually using the SCS unit hydrograph technique. Rainfall information for frequency-based floods came from the NOAA Atlas (amount areally reduced 93 percent as applicable) (NOAA 1977), SCS 24-hour type II rainfall temporal distribution and constant spatial distribution. The PMS and PMP candidate information are described in the hydrometeorologic report (NAWC 1996). The loss rate function was specified by the SCS CN technique, as previously described. The drainage basin characteristic lag defining temporal response to rainfall completes the key subbasin definition criteria. The use of the unit graph procedure and loss rate function from a single methodology (SCS procedures) as accomplished herein preserves the integrity of hydrograph computation versus mixing components of different procedures.

Hydrograph channel routes were defined between subbasin hydrograph computation points and desired hydrograph combination points listed as follows:

NF subbasin to CP subbasin
NF route plus CP subbasin to CC and DF subbasins
NF/CP route plus DF and CC to LG subbasin
NF/CP/DF/CC route plus LG subbasin to reservoir

These routes were accomplished using the Muskingum-Cunge method with the channel section described using eight horizontal/vertical point pairs. Longitudinal slopes and distances were measured from best available mapping and Manning's n values were estimated using established step-wise procedures with results checked against photographic references.

Hydrograph combination points were established at the following locations:

Routed NF subbasin plus CP subbasin
Routed NF/CP plus CC and DF
Routed NF/CP/CC/DF plus LG

A reservoir route was the final step, where needed, to route flows through either the existing or proposed reservoir. Refer to **Figure 4.1** for a line schematic of this model network.

Where necessary, changes in antecedent moisture were reflected by adjusting CNs. Baseflow from snowmelt was added as a constant as applicable and no baseflow was added for groundwater contribution as it is considered negligible.

Figure 4.1. Hydrologic network schematic.

4.8 Safety Evaluation Flood

4.8.1 General

The SEF (NAP 1985) is defined as the maximum flood which a dam has the capability to withstand without failure. Generally, and in Colorado, the SEF is defined in terms of the PMF (NAP 1983), which is the flood that may be expected from the most severe combination of critical meteorological and hydrologic conditions that are reasonably possible in the region. The full PMF is adopted as the SEF for the Elkhead Dam, as the structure is classified a large, Class I (high hazard) structure. Standardized procedures have been established to determine the PMP and its pattern within the PMS. The standard procedure for this general geographical region is described in Hydrometeorological Report No. 49 (HMR49) (NOAA 1977). These reports typically produce such large regional precipitation values that they can exceed what many people feel are "reasonably possible" events in specific geographical areas. In order to customize a PMP for the site, a site-specific PMP/PMS evaluation was conducted (NAWC 1996). This site-specific analysis used the standard HMR49 methodologies combined with updated technologies and procedures. This produced 12 PMS candidates (7 general and 5 local) through transposition of known storms to the site.

The presumption inherent in this site-specific analysis is that our recorded precipitation and streamflow history of 100 to 150 years contains an adequate base of information, which when maximized using the most severe combination of reasonably possible physical events, produces a family of maximum possible storms. Since these storms vary in amount, pattern, size, and duration, each must be hydrologically modeled under similarly maximized watershed conditions to determine which storm produces the flood with the maximum runoff rate and volume. That maximum flood is then accepted as the site-specific PMF.

4.8.2 HMR49 Probable Maximum Flood

Over the last approximately 20 years, the PMP/PMF events have been computed for this site using the published regional standard reference, HMR49 (NOAA 1977). Those evaluations establish a common frame of reference for this report.

A recent report (Harza 1991) documents the PMF for this site based on HMR49 (NOAA 1977) as an inflow flood peak of 40,600 cfs. This report was approved by the SEO on July 8 1991, and, based on communications with the SEO, is considered current. That report has until now been accepted as face value as the SEF-approved documentation without further evaluation. An earlier PMF report (CDOW 1984) provides valuable reference material, but has been superseded by the Harza report (Harza 1991). These reports were prepared in response to a judgment made from the 1979 Phase I National Dam Safety Program (Hydro-Triad 1980) inspection that the existing dam had inadequate spillway capacity to pass the PMF without overtopping.

The peak flow values produced by this method range from approximately 13,000 to 60,000 cfs depending upon variances in the meteorological/hydrologic modeling processes. Concern that most of these values are significantly higher than used for the original dam design and construction and that this may require a spillway upgrade encouraged the dam owner to initiate a site-specific PMP/PMF study. This was reinforced by the knowledge that other similar site-specific studies have generally lowered the PMF.

4.8.3 Hydrometeorologic Analysis

A separately published site-specific hydrometeorological analysis (NAWC 1996) was conducted for the Elkhead Dam/Reservoir. That analysis describes 12 site-specific PMS candidates, including precipitation, temporal distribution, and spatial distribution which produces that basis for hydrologic modeling of each of the storms. That report is well written and incorporated by reference. Please refer to this document for a comprehensive description of the PMS and PMP, as needed.

4.8.4 Probable Maximum Precipitation/Flood

The aforementioned report (NAWC 1996) provides the meteorologic (PMS candidates) basis for computation of 12 site-specific PMF candidates in basic conformance with the hydrologic modeling procedures previously described. All computed discharges represent maximum inflow flood peaks into the reservoir. A summary of the precipitation and basin characteristics for each is presented in the following paragraphs.

The first PMS candidate (Storm 1) is a general storm referred to as the "Bug Point Transposition." This is a spatial transposition of an actual, least orographic, extreme convergence storm. This storm occurred over Bug Point, Utah, on September 4-6, 1970, and was transposed to August 20. The transposed storm isohytals (rainfall spatial pattern and amounts) are shown in Figure 2.7 (NAWC 1996). The storm pattern was rotated ±20° and PMFs computed for each position. This produced an optimum flood peak discharge of 36,052 cfs. Since the storm occurred in late August, snowmelt was not considered as a contributing factor and normal soil moisture (AMC II) conditions were used in developing the runoff CNs. The storm duration/temporal distribution is that of the original storm shown in Figure 2.7a (NAWC 1996) with precipitation amounts adjusted to have the same percent of precipitation in each period.

The "Silverton Transposition" (Storm 2) is also a general storm. This is a spatial transposition of an actual, strongly orographic storm. This storm occurred south of Silverton, Colorado, on September 4-6, 1970, and was transposed to August 20. The transposed storm isohytals are shown in Figure 2.8 (NAWC 1996). Since the storm occurred in late August, snowmelt was not considered as a contributing factor and normal soil moisture AMC II conditions were used in developing the runoff CNs. The storm duration/temporal distribution is that of the original storm shown in Figure 2.8a (NAWC 1996) with precipitation amounts adjusted to have the same percent of precipitation in each period. The pattern and distribution produced an optimum flood peak discharge of 16,236 cfs.

The "Gladstone Transposition" (Storm 3) is also a general storm. This is a spatial transposition of an actual strongly orographic storm which produced heavy rains in New Mexico and southwestern Colorado with the maximum reported at Gladstone, Colorado. The storm occurred on October 4-5, 1911, and was transposed to September 15. The transposed storm isohytals are shown in Figure 2.9 (NAWC 1996). Since the storm occurred in October, snowmelt was not considered to be a contributing factor and normal soil moisture, AMC II, conditions were used in developing the runoff CNs. The storm duration/temporal distribution is that of the original storm shown in Figure 2.9a (NAWC 1996) with precipitation amounts adjusted to have the same percent of precipitation in each period. The pattern and distribution produced an optimum flood peak discharge of 13,403 cfs.

The Dinosaur Transposition" (Storm 4) is also a general storm. This is a spatial transposition of an actual extratropical cyclone storm with southerly flow followed by a cold front passage. This storm occurred from June 8-12, 1970, over the Dinosaur National Monument in Colorado, the greatest daily rainfall amount was reported on June 10, 1970 and was transposed to June 15. The transposed storm isohytals are shown in Figure 2.10 (NAWC 1996). Since the storm occurred in June, it is possible that it occurred near the peak of the snowmelt hydrograph; thus, snowmelt was considered as a contributing factor. Therefore, a 100-year baseflow of 2,500 cfs was added to reflect snowmelt runoff. The storm duration/temporal distribution is that of the original storm shown in Figure 2.10a (NAWC 1996) with precipitation amounts adjusted to have the same percent of precipitation in each period. The pattern and distribution produced an optimum flood peak discharge of 9,732 cfs.

The “Glenwood Springs Transposition” (Storm 5) is also a general storm. This is a spatial transposition of an actual extratropical cyclone storm with a southerly flow followed by a cold front passage. The storm occurred from June 25-26, 1969, over Glenwood Springs, Colorado, during which time 3.20 inches of the 3.97 inches of rainfall fell in the first 24 hours. This amount was transposed to June 5. The transposed storm isohytals are shown in Figure 2.11 (NAWC 1996). Since the storm occurred in June, it is possible that it occurred near the peak of the snowmelt hydrograph; thus, snowmelt was considered as a contributing factor. Therefore, a 100-year baseflow of 2,500 cfs was added to reflect snowmelt runoff. Low permeable soil conditions AMC III were used in developing the runoff CNs. The storm duration/temporal distribution is that of the original storm shown in Figure 2.11a (NAWC 1996) with precipitation amounts adjusted to have the same percent of precipitation in each period. The pattern and distribution produced an optimum flood of 9,476 cfs.

The “North Ogden Transposition” (Storm 11) is also a general storm. This is a spatial transposition of an actual storm which was strongly orographic, localized rainfall, produced by a number of convective cells which reportedly followed the same track in an organized manner known as an echo train. This storm occurred from September 5-10, 1991. On September 7 and 8, a 24-hour rainfall amount of 8.4 inches fell at North Ogden, Utah, of a total rainfall for the entire storm of 9.85 inches. It was transposed to August 22. The transposed storm isohytals are shown in Figure 2.12 (NAWC 1996). Since the storm occurred in September, snowmelt was not considered as a contributing factor and normal soil moisture AMC II conditions were used in developing the runoff CNs. The storm duration/temporal distribution is that of the original storm shown in Figure 2.11a (NAWC 1996), with precipitation amounts adjusted to have the same percent of precipitation in each period. The pattern and distribution produced an optimum flood of 4,874 cfs.

The “Cimarron Transposition” (Storm 12) is also a general storm. This is a spatial transformation of an actual/widespread storm with an embedded local convective storm. This storm occurred on June 3, 1952, and was transposed to that date. The transposed storm isohytels are shown in Figure 2.13 (NAWC 1996). Since the storm occurred in June, it is possible that it occurred near the peak of the snowmelt hydrograph; thus, snowmelt was considered a contributing factor. Therefore, a 100-year baseflow of 2,500 cfs was added to reflect snowmelt runoff and low permeable soil conditions AMC III were used in developing the runoff CNs. The storm duration/temporal distribution is that of the original storm shown in Figure 2.13b (NAWC 1996) with precipitation amounts adjusted to have the same percent of precipitation in each period. The pattern and distribution produced an optimum flood of 7,651 cfs.

The “Opal Transposition” (Storm 6) was a local thunderstorm. This is a spatial transformation of an actual storm which produced 7 inches of rainfall in under 2 hours. This storm occurred on August 16, 1990, and was transposed to that date. Since the storm occurred in August, snowmelt was not considered a contributing factor and normal moisture AMC II conditions were used in developing the runoff CNs. The transposed storm was configured in the shape of a 2:1 ellipse with an areas of 50 mi^2 of constant areal rainfall distribution. The temporal distribution and rainfall amounts are provided in Table 2.5 (NAWC 1996). This storm was arranged over each individual/subbasin and the runoff modeled to determine which location of the ellipse produced the highest runoff peak. If the subbasin was greater than 50 mi^2, then no ellipse was used, but rather the rainfall was adjusted to a depth equivalent of a storm located over the whole basin. From this, two storm locations produced distinctly higher results; those located over Dry Fork and Long Gulch subbasins, with the Long Gulch position producing the highest runoff rates. From the results of this analysis, it was determined that all local thunderstorms configured in this manner would be evaluated with the ellipse located over Dry Fork and Long Gulch individually, and the larger runoff producing orientation adopted. The optimum flood for this storm was with the ellipse located over Long Gulch which produced a peak flow of 29,363 cfs.

The “Morgan Transposition” (Storm 7) was also a local thunderstorm. This is a spatial transposition of an actual storm which was caused by low-level moisture being pushed over the Wasatch Range by a wave-like pulse combined with surface heating and local terrain effects. This storm occurred on August 16, 1958, near Morgan, Utah, and was transposed to that date. Since the storm occurred in August, snowmelt was not considered a contributing factor and normal AMC II conditions were used in developing runoff CNs. The transposed storm was configured in the shape of a 2:1 ellipse with an area of 50 mi^2 of constant areal rainfall distribution. The temporal distribution and rainfall amounts are provided in Table 2.6 (NAWC 1996). The storm was arranged and computation completed as described for the “Opal Transposition” storm. The optimum flood for this storm was with the ellipse located over Long Gulch with produced a peak flow of 14,886 cfs.

The “Muddy Creek Summer and Spring Transpositions” (Storms 8 and 9), respectively, were also local thunderstorms. Each is a spatial transposition of an actual storm which was caused by a synoptic pattern characterized by an upper level low over the coast, supported by high pressure over the Midwest. A short-wave trough passed north to south and a short duration thunderstorm occurred which was probably triggered and sustained by surface heating. This very recent and nearby storm was documented during the progress of work on this project. This storm occurred on June 20, 1994, over the Muddy Creek drainage basin, which is located 60 miles to the southeast on the west side of the Continental Divide in Colorado. It was transposed to two separate dates, June 5 and July 5, to reflect the possibility that this storm could have occurred in the summer or spring. For the summer (July 5) storm snowmelt was not considered a contributing factor and normal AMC II moisture conditions were used in developing runoff CNs. The transposed storm was configured in the shape of a 2:1 ellipse with an area of 50 mi^2 of constant areal rainfall distribution. The temporal distribution and rainfall amounts are provided in Table 2.7 (NAWC 1996). The storm was arranged and computations completed as described for the “Opal Transposition” storm. The optimum flood for this storm was with the ellipse located over Long Gulch which produced a peak flow of 8,698 cfs. For the spring (June 5) storm, it is possible that it occurred near the peak of the snowmelt hydrograph; thus, snowmelt was considered as a contributing factor. Therefore, a 100-year baseflow of 2,500 cfs was added to reflect snowmelt runoff and low permeable soil conditions AMC III were used in developing the runoff CNs. The transposed storm was configured in the shape of a 2:1 ellipse with an area of 50 mi^2 of constant areal rainfall distribution. The temporal distribution and rainfall amounts are provided in Table 2.8 (NAWC 1996). The storm was arranged and computations completed as described for the “Opal Transposition” storm. The optimum flood for this storm was with the ellipse located over Long Gulch, which produced a peak flow of 15,172 cfs.

The “Mesa Verde Transposition” (Storm 10) was also a local thunderstorm. This is a spatial transposition of an actual storm which occurred in a post-frontal air mass. This storm occurred on August 3, 1924, in the Mesa Verde area of Colorado and was transposed to August 15. Since the storm occurred in August, snowmelt was not considered a contributing factor and normal AMC II moisture conditions were used in developing runoff CNs. The transposed storm was configured in the shape of a 2:1 ellipse with an area of 50 mi^2 of constant areal rainfall distribution. The temporal distribution and rainfall amounts are provided in Table 2.9 (NAWC 1996). The storm was arranged and computations completed as described for the “Opal Transposition” storm. The optimum flood for this storm was with the ellipse located over Long Gulch, which produced a peak flow of 10,295 cfs.

Comparison of the results of the 12 floods which result from these 12 PMP candidate storms allows the conclusion to be reached that the “Bug Point Transposition” general storm produces both the largest peak flowrate and volume flood. The “Bug Point Transposition” is therefore adopted as the PMP/PMS and its resulting flood adopted as the PMF for use in this report. The model and results of this event are provided in **Appendix B**.

4.8.5 Design Application - Proposed Project

Several of the critical proposed project components are sized using the PMF hydrology information. These include primarily the service and emergency spillways. They will exist in relationship to each other as described in Section 2 of this report. The hydraulic characteristics of the primary outlet and spillway stilling basin are sized based on the frequency-based flood hydrology information described in this document.

The emergency spillway was the primary element sized using the PMF criteria. It was sized in conjunction with the service spilling sizing described in this document. Fortunately, an excellent saddle spillway site exists off the west end of the dam embankment. Since that saddle spatially spills into an adjacent unnamed basin, it was judged desirable to minimize the frequency of such spills to events so rare that they have not yet been experienced in recent geological time. It also enables us to configure an emergency spillway that theoretically meets safety criteria for the dam economically. It was, therefore, decided to use the 500-year rainfall flood peak flow of 5,808 cfs (attenuated to 1,734 cfs by the new project) as that event which would be handled by the project without use of the emergency spillway. The 500-year primarily snowmelt flood was not used for this purpose because of the uncertainties associated with the true attenuation provided by an enlarged reservoir, and because of the ability to reduce the peak flows by drawing down the reservoir in advance in anticipation of unusually high snowpack melt. For comparison purposes, a 100-year peak primarily snowmelt flow, on top of an already full reservoir (a conservative assumption), would fill the surcharge storage to the point of the sill of the emergency spillway. Also, for reference purposes, the estimated maximum paleoflood upstream of the dam is 1,800 to 3,300 cfs, as described in Appendix A.

The emergency spillway sill was set at 6,409 feet in order to found the structure on bedrock. A cost-effective evaluation resulted in the use of the maximum spillway width of 350 feet in the project configuration. Using these criteria, the routed PMF reaches a peak discharge of 25,506 cfs at elevation 6415.23 feet, or 3 feet below the crest of the dam. (Note that the most economical emergency spillway/principal spillway/surcharge storage volume combination remains to be optimized.) Appendix B contains the model and results of the evaluation. The relevant discharge rating for the service and emergency spillways is provided in **Table 4.3**. A stage-area-volume tabulation for the proposed reservoir is also provided in Table 4.3.

4.9 Frequency-Based Floods

4.9.1 General

Modern dams are commonly designed with an expected functional life of 50 to 100 years. While many dams last longer than 100 years, flood events less frequent than once every 500 years have little significance as design events (other than related to dam failure avoidance). Each dam component has a defined hydraulic reliability and to design each to properly function under those conditions requires knowledge of the peak flow and hydrograph of frequent flood events. Using the previously described hydrologic model, flood inflow hydrographs to the proposed reservoir have been established for both primary snowmelt and rainfall events for the 2-, 10-, 100-, and 500-year floods, as applicable.

4.9.2 Primarily Snowmelt Floods

As previously described, the flood history for the gaged streams in the Elkhead drainage basin, as represented by annual peak flows, contains no obvious rainfall-only or -dominated events. Annual peaks occur during the snowmelt period with the result that flooding is considered to be snowmelt-dominated for the period of time records which exist. Rainfall can and does occur during the snowmelt period, but available information does not reveal how significant rainfall is during snowmelt-dominated periods. A principal benefit of defining the primarily snowmelt events is to provide baseflow information at the dam associated with PMF conditions. Examination of this mixed-event history of snowmelt-dominated floods allows us to estimate flood peak flows due to snowmelt up to and including the 100-year event.

This was accomplished by several methods, including the statistical analysis previously described. The following simplifying assumptions were made from best available information in order to compute snowmelt peaks.

Table 4.3. Spillways Hydraulic Rating Information.

1. The 7-day mean flow reasonably represents the runoff due to snowmelt for the annual period of April through June.

2. Computing ratios of daily mean to 7-day mean for selected representative years of the gaged records results in a value of 1.2.

3. Computing ratios of daily peak to daily mean for selected representative years of the gaged record results in a value of 1.2 reflecting the diurnal fluctuation of snowmelt runoff. The impact of the existing dam (and proposed dam) is to attenuate the daily peak values toward the mean daily values.

Using this information, the following computations were completed:

1. The gages analysis extended to the dam site results in a mean annual peak flow of 1,250 cfs which is attenuated to 1,050 cfs by the reservoir.

2. The 100-year event was computed by four techniques shown in **Table 4.4**, and a value of 2,500 cfs, which is attenuated to 2,100 cfs by the reservoir, was chosen.

Table 4.4. Primarily Snowmelt Flood Peak Flow Rates.

Source	Q_{100} (cfs)	Q_{maf} (cfs)
Upstream gages statistical analysis extended to site	2,665	1,230
Pitlick (Pitlick 1988) Figure 3.4	2,500	NA
Wolford Hydrology (Boyle 1991, 1995) adjusted by drainage area ratio	2,890	1,100
WRI 85-4086 (Kircher et al. 1985)	1,655	616

A log-probability plot provides a 10-year inflow value of 1,800 cfs.

4.9.3 Primarily Rainfall Floods

As previously described, obvious rainfall floods are virtually absent from the period of record. The best analytical technique to estimate the peak flow and hydrograph associated with rainfall is the design storm method where runoff is computed from rainfall. Available information (NAWC 1996) indicates that rainfall can produce significant floods in the region of Colorado, but is not clear at what frequency event rainfall may be the dominant cause of flooding. The design storm method using the SCS unit graph and runoff computation procedure results in the reservoir inflow peak values presented in **Table 4.5**. An example of the design storm model and results for the 100-year event is provided in **Appendix C**.

Table 4.5. Rainfall Flood Peak Flow Rates.

Rainfall/Flood Frequency (-year)	Peak Flow Rate (cfs)

2	21
10	425
100	2,850
500	5,808

The peak for the 100-year event was used as the basis for comparing the design storm model results with results from a regional peak flow method. The model results compare well with the regional analysis (CWCB 1976) available for this area which produces a value of 2,800 cfs. There is no similar, convenient check for the resulting hydrograph volume and shape.

Given that the gage analysis reflects a site dominated by primarily snowmelt events, rather than rainfall-only events, it is interesting to note that the 100-year primarily snowmelt event peak is 2,500 cfs, which is also very close to the model results. Both the primarily snowmelt and rainfall-only frequent events are shown in the frequency discharge relationship in **Figure 4.2**.

4.9.4 Design Application - Proposed Project

Several of the critical proposed project components are sized using the frequency-based flood hydrology information. Those include the primary outlet, service spillway, emergency spillway, and the service spillway stilling basin. They will exist in the relationship of each other as described in Section 2 of this report. Note that only the sizing to meet flood criteria is described herein; sizing to satisfy other functions of these features may result in increasing or otherwise modifying the structure configuration.

The primary outlet will be sized to pass approximately the mean annual flood event without the use of the service spillway. This means it will pass a 1,050 cfs primarily snowmelt (including diurnal attenuation) flow peak with the reservoir water surface at or below 6406 feet. This feature must also be able to pass the 10-year peak flow event during construction-phase water handling. Since the primary outlet will not have to handle snowmelt runoff by itself during construction, the critical construction-phase flow is the unattenuated 425 cfs peak flow of the 10-year rainfall-only event with the water surface at ±6345 feet.

The service spillway will be sized to pass a full range of flows from 0 (in the case of no flow through the primary outlet) to approximately 10,000 cfs when assisting the emergency spillway to pass the PMF. The service spillway was sized in conjunction with the emergency spillway to allow the dam to be constructed to its maximum economic height while using a natural saddle spillway (versus a larger service spillway or another structural spillway) to handle the PMF. Setting the emergency spillway in bedrock at a crest of 6409 feet at the maximum width of 350 feet allowed by site constraints (refer to the proposed project description of the emergency spillway configuration, Section 2.2) resulted in a service spillway width of 100 feet and a crest elevation of 6406 feet to pass the 500-year rainfall flood without using the emergency spillway (note that the most economical service spillway width/surcharge storage volume combination remains to be optimized). The principal spillway alone passes the inflow 500-year flood peak of 5,808 cfs, attenuating it to 1,734 cfs through reservoir storage without use of the emergency spillway. The relevant discharge rating for the service spillway is provided in Table 4.3.

Figure 4.2. Reservoir influent flood peaks.

The stilling basin located at the base of the service spillway must operate efficiently (without significant structural damage) for a wide range of flows. It must be designed to handle at least the 100-year frequency flow event. In this case, the 100-year routed rainfall flood is attenuated from approximately 2,850 to 855 cfs by storage in the reservoir. However, the 100-year routed snowmelt flood is not attenuated by the reservoir significantly because the broad-based ascending limb of the hydrograph fills available flood storage. The 100-year snowmelt flow is estimated at 2,100 cfs which is the approximate service spillway capacity with the water level at the crest of the emergency spillway. The service spillway, which delivers water to the stilling basin, is designed with a width and wall height to withstand a portion of the routed discharge associated with the PMF event (10,095 cfs) in order to avoid wall overtopping and prevent embankment erosion. The stilling basin was not designed for the PMF event because doing so would result in a tremendously large and expensive basin and there are more cost-effective means for providing adequate erosion protection at the toe of the dam. The stilling basin is designed to handle a flow of 3,200 cfs which is approximately equal to a 1,000 year primarily snowmelt or routed rainfall return period. In the event that this design is exceeded (less frequent events occur), and flow begins to affect the proper functioning of the stilling basin, the spillway and stilling basin have been located in the left abutment to protect the dam toe from severe erosion. As an extra precaution, the dam toe to the west of the stilling basin will be armored with riprap and a sacrificial buffer berm to restrict vortex flows will be constructed as shown in Figure 2.1.

The emergency spillway is not sized to handle a specific flood, but to convey all floods exceeding the 500-year event up to and including the PMF as described in this report. The service spillway operates in conjunction with the emergency spillway to satisfy this need as described herein.

4.10 Paleofloods

Frequency-based floods which are used to size the routinely operable elements of dams, while occasionally rare, are events which can reasonably be expected to occur during the 50- to 100-year life of a project. We can also be reasonably confident in the peak flow rates and other characteristics of these events based upon knowledge of and extensions to recorded information, some of which dates to the middle of the 19th century.

Unfortunately, the PMF has no defined frequency and intentionally is such a rare, theoretical event that it lies outside the expected time period of human experience. In addition, it constitutes a physical event which is difficult to imagine. To help place the magnitude of this type of event in perspective, a paleoflood evaluation was completed. This local and regional analysis examines ancient geologic, biologic, and other evidence of floods, which is available for events which have occurred during the current geologic period, the Holocene (since glaciation, 10,000 to 14,000 years ago), but oftentimes, lie outside the period of human records. That study is included in Appendix A.

As can be seen from that report, the largest flow in that time period, 1,800 to 3,300 cfs (probably occurring in 1984), is approximately equivalent to the 100-year event. That is, the PMF peak flow as estimated herein is 10 times the largest flood experienced in the last 10,000 years.

Another important point to be made is that the PMF is expected to be produced by a summer time general storm, a storm type which has produced no recorded annual peak flows in the basin nor was responsible for the maximum paleoflood, but is still theoretically possible.

5. REFERENCES

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TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

EXECUTIVE SUMMARY

Elkhead Dam/Reservoir is located on Elkhead Creek approximately 3 miles upstream of its confluence with the Yampa River in Moffat and Routt Counties in northwest Colorado, approximately 8 miles northeast of Craig. Two prior reports (Hydrosphere 1993, 1995) have provided documentation which recommends enlarging the existing dam/reservoir. Specifically, the dam will be raised to a height of 130 feet, making it a Class I "large" structure by Colorado State Engineer's Office (SEO) standards. This report presents a detailed hydrology study supporting the design of the enlarged facility. In addition, prior probable maximum flood (PMF) studies have indicated that the existing dam will not handle the PMF without overtopping. The enlarged facility will also incorporate facilities to satisfactorily handle the PMF.

The enlarged facility will contain 4 primary components: (1) the existing dam will be raised 43.5 feet to a crest elevation of 6418.5 feet, (2) a new primary outlet will be constructed, (3) a new service spillway will be constructed, and (4) an emergency spillway will be added. The dam will be raised at its existing location, including a new road alignment. The primary outlet will handle all releases up to the mean annual peak flood, and will be the main flow handling structure. The service spillway will handle any flood above the mean annual peak flood, up to the 100- year event. The flood surcharge storage and emergency spillway will assist in passing floods more infrequent than once in 100 years. The primary outlet and service spillway will be constructed at new locations. The emergency spillway will be constructed through an existing topographic saddle.

In order to size these facilities, two related hydrologic analyses were completed. In order to meet State of Colorado dam safety standards, this enlarged dam must be able to withstand without failure by overtopping, the inflow design flood (IDF) produced by the statistically undefined probable maximum precipitation (PMP). This is also known as the safety evaluation flood (SEF). In addition to being safe from failure, the dam must function properly under flood conditions which can reasonably be expected to occur during its design life. In order to properly size the individual facilities needed to handle more frequent flood events, the 2-, 10-, 100-, and 500-year frequency-based flood events were also computed.

Hydrologic computations using computer model HEC-1 (USACOE 1990) and statistical and regional analyses were utilized to estimate the peak discharge rates and volumes for these flood events. The 205-square-mile drainage area was divided into 5 subbasins with connecting conveyance elements as the basic model configuration. Information about soils/geology, vegetation, topography, land use, meteorology, and streamflow were mathematically described for each subbasin as model input. A sensitivity analysis of 3 design storm methods resulted in the choice of the SCS unit graph and design storm method as the computational procedure.

It was believed that for this site, a PMP calculated using standard regional information would result in an unreasonably high SEF. Therefore, a site-specific PMP/PMS evaluation was conducted by North American Weather Consultants. This separately published site-specific hydrometeorologic analysis produced 12 transposed, maximized storms as candidates for adoption as the PMP. Each storm was hydrologically modeled by flood to determine which produced the most stress. The precipitation associated with that most stressing flood was then adopted as the PMP. This analysis concluded that the "Bug Point Transposition" general storm which produces a peak inflow flood of 36,052 cfs, is the site-specific PMP/PMS. For practical reference purposes, the maximum paleoflood (largest flood which has occurred in the current geologic period of 10,000 years) was determined from paleo evidence to be from 1,800 to 3,300 cfs. The produced "Bug Point Transposition" PMP and the IDF was then utilized to size and configured the service spillway, flood surcharge storage, and emergency spillway.

Frequent flood events can be caused by primarily snowmelt or rainfall events depending on the time of the year. Local meteorologic and stream gage information is limited spatially and temporary. The stream gage information indicates a very clear evidence of snowmelt- dominated high flows. Using this information, the 100-year reservoir inflow peak flow rate was computed to be 2,500 cfs. While rainfall alone rarely has caused significant flows to the reservoir, peak flows were estimated using the design storm concept and the same basic model as was used to compute the SEF. The resulting peak flow rates are shown in the table below.

Rainfall Flood Peak Flow Rates	
Frequency (yr)	Peak Flow Rate (cfs)
2	21
10	425
100	2,850
500	5,800

These values are consistent with the flow records and regional knowledge of flood events that indicate most flood peaks up to the 100-year event are snowmelt-dominated and less frequent events, rainfall-dominated. These values were used to size hydraulic facilities needed to routinely handle water, including the primary outlet, stilling basin, service spillway, and construction stage water handling.

Table 4.3. Spillways Hydraulic Rating Information.

Spillways Rating

Stage	Depth (ft)	Service Spillway Average Length (ft)	Q (cfs)	Depth (ft)	Emergency Spillway Average Length (ft)	Q (cfs)
6406.00*	0	100	0	0	350	0
6406.25	0.25	100	45	0	350	0
6406.50	0.5	100	127	0	350	0
6406.75	0.75	100	234	0	350	0
6407.00	1	100	360	0	350	0
6407.50	1.5	100	661	0	350	0
6408.00	2	100	1,018	0	350	0
6409.00**	3	100	1,871	0	350	0
6409.25	3.25	100	2,109	0.25	350.5	118
6409.50	3.5	100	2,357	0.5	351	335
6409.75	3.75	100	2,614	0.75	351.5	616
6410.00	4	100	2,880	1	352	950
6410.50	4.5	100	3,437	1.5	353	1,751
6411.00	5	100	4,025	2	354	2,703
6412.00	6	100	5,291	3	356	4,995
6413.00	7	100	6,667	4	358	7,733
6414.00	8	100	8,146	5	360	10,867
6415.00	9	100	9,720	6	362	14,365
6417.00	11	100	13,134	8	366	22,360
6419.00	13	100	16,874	10	370	31,591

The equations corresponding to these discharges are: General: $Q = CLH^{3/2}$ Service Spillway: $Q = 3.6\ (100)\ H^{3/2}$ Emergency Spillway: Q = 2.7 (350

* Service spillway crest, storage volume - 44899 ac-ft ** Emergency spillway crest, storage volume = 47700 ac-ft

Reservoir Area and Storage Capacity

Stage (ft)	Average Area (ac)	Total Volume (ac-ft)
6295	0	0
6300	2.12	4.7
6310	26.89	274
6320	81.5	1089
6330	151.5	2604
6340	244	4816
6350	299	7806
6360	402	11607
6370	497.15	16310
6380	640.5	22451
6390	791.5	29961
6400	931.55	38841
6406	1009.55	44899
6409	1070.25	47700
6410	1090.5	49261
6414	1158.5	53895
6418	1220.5	58777
6418.5	1231.25	59409